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The Application of Traffic Simulation Model to Predict Initiation and Progression of Crack for Flexible Pavements

Sugeng Wiyono[a,*]

[a] *Department of Civil Engineering, Islamic University of Riau, Km 10 Kaharudin Nasution Road, Pekanbaru, Riau, Indonesia*

Abstract

In the life-cycle prediction of road pavement, it needs the model that should be able to predict the expected change of condition in the future. The model should consider current condition, pavement strength and age characteristics, environment, incremental time and incremental traffic. The aim of this study is to application the traffic simulation model for predicting initiation and progression of crack on road pavement. The aim of the study can be achieved by developing a computer simulation model that can predict road deterioration. The research develop coefficient of each models that agree with local condition based on observed data that collected for 1.5 years. These models are able to predict progression of cracking with R^2=0.5925 to 0.8765 more appropriate than the existing model (R^2=0.304 to 0.314). The coefficient of crack initiation model has difference with the existing models that are 5.7% to 20% for asphalt mix on asphalt pavement, 2.8% to 14% for asphalt mix on stabilized base, 1.6% to 2.2% for asphalt mix on granular base. While progression of cracking are 5.7% to 20% for asphalt mix on asphalt pavement, 2.8% up to 14% for asphalt mix on stabilized base, 1.6% to 2.2% for asphalt mix on granular base. In addition, the cracking model can be used as guidance for maintenance intervention criteria.

Keywords: traffic simulation, crack initiation, crack progression

1. Introduction

The deterioration of paved roads is defined by the damage trend of its surface condition over time. The

* Corresponding author.
E-mail address: sugenguir@gmail.com.

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defects of a pavement surface, which is usually quantified through a pavement condition survey, are classified under three major models of distress, namely; cracking, disintegration, and permanent deformation. The main focus of this paper is on the crack damages because cracking often triggers the application of maintenance treatments and cracking can be the decisive factor in determining the most appropriate rehabilitation option among others.

Cracking is perhaps one the most important distresses in bituminous pavements. The development of cracking is considered directly in most mechanistic design procedures and indirectly in most empirical design procedures. A primary bituminous pavement design objective is to minimize cracking. Cracking is a distress that is readily identifiable and universally acknowledged as a sign of pavement deterioration. However, the modeling of cracking is quite complex. There are many factors that can affect the development of cracks, and once present, the proliferation of cracking may be affected by the same factors, probably the different factor with the other, or a combination of both.

There are many different approaches to analyze and understand bituminous cracking. Cracks may be defined by their shape (e.g., crocodile, block, and linear), their primary causative action (e.g., fatigue, reflection, and thermal), and their location on the pavement (e.g., edge, wheel path). Cracks are often described by the manner they develop (e.g., top-down or bottom-up) and the life of the pavement when the cracks develop. This paper considers empirical models developed by Paterson (1987) and Bennett *et al* (1995) for three types of crack progressions, i.e. the time of initiation of structural crack, transversal or thermal crack and structural cracks.

The time of initiation of structural crack, measure in years, as suggested by Paterson (1987), is in the form of Eq. (1), i.e.

$$ICX = K_{icx} a_0 e^{\left(a_1 YE4 / SNC^2\right)} \tag{1}$$

where *ICX* is the time of initiation of structural cracking (in years), K_{icx} is the structural cracking initiation factor, *YE4* is the annual number of equivalent standard axle loads (ESAL), *SNC* is the modified structural number of the pavement and, a_0 and a_1 are the calibration parameters.

For the prediction of spacing between thermal cracks, measure in meter, Simpson *et al* (1995) suggested a model as shown in Eqn. 2.

$$CRKSPACE = 0.305 AGE2^{a_0} 10^{a_1} \tag{2}$$

where *CRKSPACE* is the spacing between thermal cracks (in meter), *AGE2* is the age of the pavement surface, and a_0 and a_1 are the calibration parameters.

Equation 3 predict the spacing of transverse cracks CW m long, where CW is the full paved width of the pavement. HDM-4 will express thermal cracking in linear m per km, and ACT is used to express this, as is shown in Eq. (3):

$$ACT = \frac{\mathrm{CW1000}}{CRKSPACE} \tag{3}$$

where *AGE2* is the age of the pavement surface and, a_0 and a_1 are the calibration parameters. The third model considers in this paper is the prediction of the incremental area of crack as given in Eq. (4).

$$CRX_t = (1-z)50 + z\left[z a_0^2 Neci + z 0.5^{a_1} + (1-z)50^{a_2} \right]^{\frac{1}{a_2}} \tag{4}$$

where CRX_t is the incremental area of cracking at time *t* (this study), z is the simoidal model parameter, *NEci* is the cumulative ESA since cracking initiation and, a_0, a_1 and a_2 are the calibration parameters. Paterson *et al* (1987) suggests that z=1 if TCI<t_{50}; otherwise z=–1, $a_0^1 = a_0$ SNC^{a1}(this study). TCI is the time since cracking initiation and t_{50} is 50^{a2} - $0.5^{a2}/a_0^1$ a_2; i.e., time to 50 per cent area cracked (this study).

The models given in Eqs. (1)–(4) are used in the HDM–IV manual. Because the basis of the data used to develop the models is different from the Indonesian roads in terms of traffic loading and climate, it is believed that the models are not directly applicable to the analysis of Indonesian highways. Therefore, there is a need to define the parameters that are directly associated with the local environments. In this study, the calibration parameters a_0, a_1 and a_2 in the corresponding models are derived using a traffic simulation model developed and empirical data collected for the environments in Indonesia. The study focused on three types of pavements, i.e. Asphalt Mix on Asphalt Pavement (AMAP), Asphalt Mix on Stabilized Base (AMSB) and Asphalt Mix on Granular Base (AMGB).

2. Metodologi

2.1. Work program

This study is based on systematic approach in order to achieve such an aim. Fig.1 shows the outline of the methodology adopted in the study.

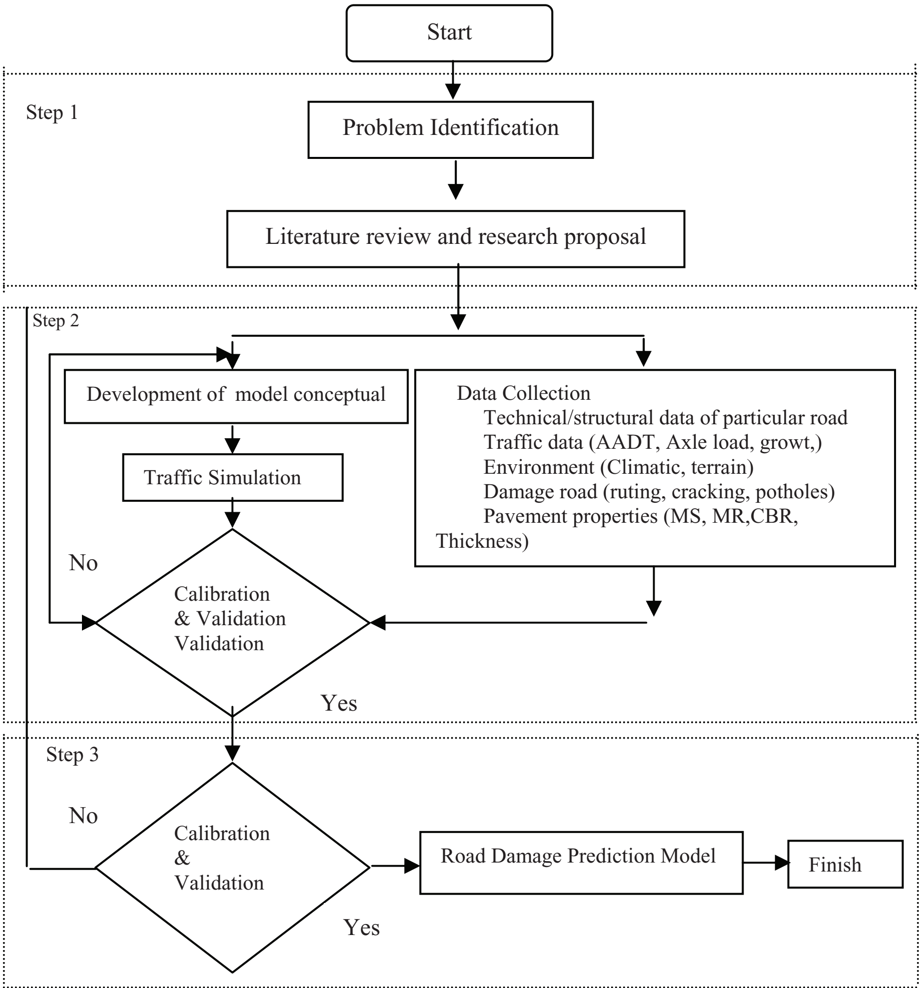

Fig. 1. Work program

2.2. Field data collection procedure

Two sections of main trunk roads in Riau Province, Sumatera, i.e., Kandis Section (5 km long) and Sorek Section (3 km long), were used for the collection of information pertaining to the analysis of cumulative standard axles and pavement surface distresses. Each section was subdivided into a subsection of 1km long.

The data for each section of the roads was collected three times at an interval of six months. This means that traffic and pavement distresses on all sections were monitored over a period of 18 months. The data gathered during each collection exercise were the traffic characteristics, progression of crack intensities, progression of rutting and potholes, surface temperature, mean monthly precipitation, and pavement deflection using a Benkelman Beam instrument. The pavement structural number is computed using the equation suggested by Paterson (1987).

The pavement where crack existed was cored to evaluate crack mechanisms as well as to establish mix properties. The California Bearing Ratio of the sub–grade was determined using the DCP method.

3. Results

In terms of traffic characteristics, both sections of the roads are considered as heavily loaded with traffic since the average ESA was greater than 0.6 million per lane per year or about 1666 ESA per lane per day (Paterson et. al 1987). In general, traffic composition was dominated by commercial vehicles which contributed to about 57% – 63% of the daily traffic. The temperatures of the pavements and atmosphere during data collections were in range of 40°C–55°C and 30°C–33°C, respectively. The average monthly rainfall was 40 mm – 125 mm and the category of the environment tropic region and wet no freezes.

Based the simulation results and data collected, the parameters associated with crack prediction models as given by Eqns. 1–3 that reflect the local environments are summarized in Tables 1–4, respectively. All variables are as defined earlier. The models developed in this study were validated using a new set of data and the respective R^2–values are tabulated in Table 5. Based on these R^2–values, it may be inferred that calibration parameters derived suit the Indonesian environments for asphalt pavement crack analysis

Table 1. Tabulation of data observed from the field and calculated values for comparisons

Time	Type of pavement	ICX (year)		ACT (m/1000m)		CRX (%)	
		Measured	Paterson (1987)	Measured	Simpson *et al* (1994)	Measured	Paterson (1987)
First Time	AMAP (Kandis Section)	1.5	3.7182	0.4950	0.4421	0	0
SecondTime				0.6091	0.6579	1.78095	1.78090
Third Time				1.0104	1.4502	3.2746	23.6307
First Time	AMSB (sorek Section)	2.5	3.7182	0.06964	0.14914	1.00310	1.18797
SecondTime				0.20714	0.22363	3.24964	19.0242
Third Time				0.27767	0.31191	6.64695	86.5511
First Time	AMGB (Sorek Section)	3.5	3.7182	0.16888	0.22306	0.86850	0.68509
SecondTime				0.36520	0.75674	3.87280	7.72444
Third Time				0.59539	0.93366	5.48571	41.8153

Table 2. Parameters for structural cracking initiation

Model	$ICX = K_{icx} a_0 e^{\left(a_1 yE4 / SNC^2\right)}$					
Pavement Type	*A*$_0$		*a*$_1$		Kicx	
	This study	Paterson (1987)	This study	Paterson (1987)	This study	Paterson (1987)
AMAP	9.48	8.61	-25.8	-24.4	0.75 to 2	0.5 to 2.5
AMSB	9.17	8.61	-25.1	-24.4	0.43	0.5 to 1.3
AMGB	8.8	8.61	-24.8	-24.4	0.49	-

Table 3. Parameters for transversal or thermal crack

Model	$CRKSPACE = 0.305 AGE2^{a_0} 10^{a_1}$			
Climatic Zone	Value of a_0		Value of a_1	
	This study	Simpson *et al.,* (1994)	This study	Simpson *et al.,* (1994)
Wet-No Freeze	-0.571 to 0.856	-1.12	3.23 to 3.89	-

Table 4. Parameters for predicting structural crack progression

Model	$CRX_t = (1-z)50 + z\left[z a_0^2 Neci + z0.5^{a_1} + (1-z)50^{a_2} \right]^{\frac{1}{a_2}}$									
Type of Pavement	Parameters						Model Statistics			
	a_0		a_1		a_2		CV (%)		r^2	
	This Study	Paterson (1987)	This Study	Paterson (1987)	This Study	Paterson (1987)	This Study	Paterson (1987)	This Study	Paterson (1987)
AMAP	1895	3330	-5.22	-4.25	0.27	0.25	4.3	54	0.845	0.314
AMGB	2855	3330	-4.65	-4.25	0.26	0.25	0.23	54	0.593	0.314
AMSB	2650	3330	-4.24	-4.25	0.25	0.25	0.28	48	0.816	0.304

Table 5. R^2–values for Structural Crack and Thermal Crack Progression Models

Type of pavement	R^2 Value for Structural Crack	R^2 Value for Thermal Crack
AMAP New	0.8765	
AMAP Existing	0.8264	0.823
AMGB	0.5925	0.879
AMSB	0.8158	0.751

4. Concluding remarks

The findings of this study may be summarized as follows:

(i) In general, each calibration parameter set in the models used in HDM–IV is constant for all types of pavements. However, this is not the case for the Indonesian highways. The result shows that each calibration parameter varies with types of pavement.

(ii) Compared with the default values given in the HDM–IV, for crack initiation, the difference in the coefficients is in the range of 5.7% to 20% for asphalt mix on asphalt pavement, 2.8% to 16.2% for asphalt mix on stabilization base and 1.6% to 2.2% for asphalt mix on granular base. In crack progression model, the difference in the coefficients is in the range of 8.0% to 75% for asphalt mix on asphalt pavement, 4.0% to 16.6% for asphalt mix on stabilization base and 0% to 25.6% for asphalt mix on granular base.

(iii) The models developed are able to predict the progression of cracks with reasonably good accuracy, i.e the R^2–values for the models are in the range of 0.59 to 0.88.

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